

CECW-ED

**DEPARTMENT OF THE ARMY  
U.S. Army Corps of Engineers  
Washington, DC 20314-1000**

EM 1110-2-2105  
Change 1

Manual  
No. 1110-2-2105

31 May 1994

**Engineering and Design  
DESIGN OF HYDRAULIC STEEL  
STRUCTURES**

1. This Change 1 to EM-1110-2-2105, 31 March 1993, updates Appendix H.
2. Substitute the attached pages as shown below:

Remove page

Insert page

ii

ii

A-1 and A-2

A-1 and A-2

H-1

H-1

3. File this change sheet in front of the publication for reference purposes.

FOR THE COMMANDER:



WILLIAM D. BROWN  
Colonel, Corps of Engineers  
Chief of Staff

DEPARTMENT OF THE ARMY  
U.S. ARMY CORPS OF ENGINEERS  
Washington, DC 20314-1000

EM 1110-2-2105

CECW-ED

Manual  
No. 1110-2-2105

31 March 1993

Engineering and Design  
DESIGN OF HYDRAULIC STEEL  
STRUCTURES

**1. Purpose.** This manual prescribes guidance for designing hydraulic steel structures (HSS) by load and resistance factor design (LRFD) and guidance for fracture control. Allowable stress design (ASD) guidance is provided as an alternative design procedure or for those structure types where LRFD criteria have yet to be developed.

**2. Applicability.** This manual applies to HQUSACE/OCE elements, major subordinate commands, districts, laboratories, and field operating activities having responsibility for design of civil works projects.

FOR THE COMMANDER:



WILLIAM D. BROWN  
Colonel, Corps of Engineers  
Chief of Staff

**DEPARTMENT OF THE ARMY  
U.S. ARMY CORPS OF ENGINEERS  
Washington, DC 20314-1000**

EM 1110-2-2105

CECW-ED

Manual  
No. 1110-2-2105

31 March 1993

**Engineering and Design  
DESIGN OF HYDRAULIC STEEL  
STRUCTURES**

**Table of Contents**

<b>Subject</b>	<b>Paragraph</b>	<b>Page</b>	<b>Subject</b>	<b>Paragraph</b>	<b>Page</b>
<b>Chapter 1</b>					
<b>Introduction</b>			Commentary on Paragraph 3-4, Reliability Factors for HSS . . . . .	3-8	3-2
Purpose . . . . .	1-1	1-1	Commentary on Paragraph 3-6, Fatigue and Fracture Control . . . . .	3-9	3-3
Applicability . . . . .	1-2	1-1			
References . . . . .	1-3	1-1	<b>Chapter 4</b>		
Background . . . . .	1-4	1-1	<b>Allowable Stress Design</b>		
Commentary on Paragraph 1-4, Background . . . . .	1-5	1-1	General . . . . .	4-1	4-1
			Design Basis . . . . .	4-2	4-1
<b>Chapter 2</b>			Load and Stress Requirements . . . . .	4-3	4-1
<b>General Considerations</b>			HSS Types: Modifications for		
Limit States . . . . .	2-1	2-1	Allowable Stresses . . . . .	4-4	4-1
Corrosion . . . . .	2-2	2-1	Serviceability Requirements . . . . .	4-5	4-1
Dynamic Loading . . . . .	2-3	2-1	Fatigue and Fracture Control . . . . .	4-6	4-1
Inspection and Maintenance . . . . .	2-4	2-1	Commentary on Paragraph 4-3, Load and Stress Requirements . . . . .	4-7	4-2
Deviations from Prescribed			Commentary on Paragraph 4-4, HSS Types: Modifications for Allowable		
Design . . . . .	2-5	2-1	Stresses . . . . .	4-8	4-2
Commentary on Paragraph 2-2, Corrosion . . . . .	2-6	2-1			
Commentary on Paragraph 2-3, Dynamic Loading . . . . .	2-7	2-2	<b>Chapter 5</b>		
			<b>Connections and Details</b>		
<b>Chapter 3</b>			General . . . . .	5-1	5-1
<b>Load and Resistance Factor Design</b>			Design Considerations . . . . .	5-2	5-1
General . . . . .	3-1	3-1	Bolted Connections . . . . .	5-3	5-1
Design Basis . . . . .	3-2	3-1	Welded Connections . . . . .	5-4	5-1
Strength Requirements . . . . .	3-3	3-1	Commentary on Paragraph 5-1, General . . . . .	5-5	5-1
Reliability Factors for HSS . . . . .	3-4	3-1	Commentary on Paragraph 5-2, Design Considerations . . . . .	5-6	5-2
Serviceability Requirements . . . . .	3-5	3-1			
Fatigue and Fracture Control . . . . .	3-6	3-2			
Commentary on Paragraph 3-2, Design Basis . . . . .	3-7	3-2			

**EM 1110-2-2105**  
**Change 1**

**31 May 94**

<b>Subject</b>	<b>Paragraph</b>	<b>Page</b>
Commentary on Paragraph 5-3, Bolted Connections . . . . .	5-7	5-2
Commentary on Paragraph 5-4, Welded Connections . . . . .	5-8	5-2

**Appendix A**  
**References**

**Appendix B**  
**Load and Resistance Factor Design**  
**Criteria for Miter Gates**

**Appendix C**  
**Tainter Gates**

**Appendix D**  
**Tainter Valves**

**Appendix E**  
**Bulkheads and Stoplogs**

**Appendix F**  
**Vertical Lift Gates (Lock and Crest)**

**Appendix G**  
**Hydroelectric and Pumping Plants**

**\* Appendix H**  
**Flood Closure Structures**

**Appendix I**  
**Miscellaneous Hydraulic Steel Structures**

# List of Figures

Figure	Page	Figure	Page
B-1. Point load impact for miter gate girders . . . . .	B-3	B-6. Nomenclature for skin plate design . . . . .	B-11
B-2. Assumptions for intercostal end connections . . . . .	B-5	B-7. Sample intercostal section . . . . .	B-12
B-3. Nomenclature and assumed load area for intercostal design . . . . .	B-6	B-8. Girder hydrostatic loading and reactions . . . . .	B-14
B-4. Vertical cross section for example miter gate . . . . .	B-8	B-9. Sample girder cross section . . . . .	B-14
B-5. Example miter gate loading . . . . .	B-9	B-10. Example miter leaf torsion loads . . . . .	B-19

## Chapter 1 Introduction

### 1-1. Purpose

This manual prescribes guidance for (a) designing hydraulic steel structures (HSS) by load and resistance factor design (LRFD) and (b) fracture control. Allowable stress design (ASD) guidance is provided as an alternative design procedure or for those structure types where LRFD criteria have yet to be developed.

### 1-2. Applicability

This manual applies to HQUSACE/OCE elements, major subordinate commands, districts, laboratories, and field operating activities having responsibility for design of civil works projects.

### 1-3. References

References are listed in Appendix A.

### 1-4. Background

*a. Types of HSS.* Typical HSS are lock gates, tainter gates, tainter valves, bulkheads and stoplogs, vertical lift gates, components of hydroelectric and pumping plants, and miscellaneous structures such as lock wall accessories, local flood protection gates, and outlet works gates. HSS may be subject to submergence, wave action, hydraulic hammer, cavitation, impact, corrosion, and severe climatic conditions.

*b. Types of steels.* Structural grade steels used for design of HSS are as referred to in CW-05502 and American Institute of Steel Construction (AISC) (1986, 1989). High-strength structural steels may be considered where economy, simplicity of detail, or greater safety of design may result from their use. Instability, local buckling, and deflection of members shall be checked regardless of the type of steel used to fabricate the structure. However, these design limit states will generally be more critical for structures fabricated from high-strength steel.

*c. Design policy.* Previously, in accordance with EM 1110-1-2101, ASD criteria were specified for design of all HSS. LRFD is now the preferred method of design

and should be used for those structure types for which LRFD guidance is provided (see Appendixes B through I). For HSS where LRFD has been developed, ASD may be used as an alternative design method only with prior approval of CECW-ED. Chapter 4 includes ASD criteria which are required for those HSS where LRFD has not yet been developed. For design of a structure, LRFD and ASD methods shall not be combined; however, use of LRFD and ASD methods for the design of separate structures on large construction projects is allowed.

*d. Structures other than HSS.* Designs for aluminum, timber, and masonry structures, service bridges and highway structures, building construction, cold-formed steel construction, railroad bridges and other railroad structures, and open-web steel joist construction shall conform to the respective industry standards and are not included in this manual.

### 1-5. Commentary on Paragraph 1-4, Background

Historically, the ASD method has yielded safe and reliable structures; however, the method does not recognize differing variability of different load effects (live load, dead load) and resistances (i.e. bending capacity, shear capacity, fracture, etc.). For this reason, LRFD is the preferred method of design. In the ASD method, an elastic analysis is performed for the structure of interest and the computed stress is compared with an allowable stress. The allowable stress is the yield stress, buckling stress, etc., divided by a single factor of safety (FS). In order to obtain structures with a more uniform reliability and to achieve economy, a limit states design (LSD) approach such as LRFD has been adopted by most specification writing committees. The Load and Resistance Factor Design (LRFD) approach (an LSD approach) recognizes that the loads applied to a structure and resistances of structural members are random quantities. The LRFD method has two main advantages over the ASD method. First, in a limit state analysis, one does not have to assume linearity between load and force, or force and stress. Second, multiple load factors can be used to reflect the degree of uncertainty for different loads (dead, live), while application of multiple resistance factors reflects differing uncertainties in a particular resistance (bending capacity, shear capacity, etc.). Due to these advantages of LRFD, more uniform reliability is attained in the design process and in many cases a more economical structure results.

## Chapter 2 General Considerations

### 2-1. Limit States

All possible modes of failure should be considered when designing HSS. Possible failure modes are: general yielding or excessive plastic deformation, buckling or general instability, subcritical crack growth leading to loss of cross section or unstable crack growth, and unstable crack extension leading to failure of a member. The first two failure modes (general yielding and buckling) are addressed by LRFD and ASD principles while the third failure mode (fatigue) and the fourth (brittle fracture) can be addressed using fatigue and fracture mechanics principles.

### 2-2. Corrosion

*a. Introduction.* Painting is the primary method of preventing corrosion. It may be supplemented with cathodic protection in severe environments or when other design considerations so dictate. Design considerations for reducing corrosion problems include:

(1) In certain cases, very severe environments may warrant an additional thickness added to critical structural members.

(2) In general, welded connections are more resistant to corrosion than bolted connections.

(3) Intermittent welds are more susceptible to corrosion than are continuous welds.

CW-09940, CW-16643, and EM 1110-2-3400 provide guidance for preventing corrosion.

*b. Requirements.* The structural engineer shall consider corrosion effects throughout the design process. Items to consider when designing the HSS include:

(1) Detail the members as much as possible so there is access for a sandblasting hose (2-ft minimum bend).

(2) Make provisions for sand to escape where member connections form open-ended chambers.

(3) Try to avoid lap joints but where used, seal weld the joint.

(4) Grind slag, weld splatter, or any other deposits off the steel.

(5) Where dissimilar metals are used select the proper material as recommended by Kumar and Odeh (1989), avoid large cathode-to-anode area ratios, use isolators, and paint both surfaces.

### 2-3. Dynamic Loading

HSS are often subjected to unpredictable dynamic loading due to hydraulic flow. Where dynamic loading is known to exist, but the loading function is not defined, ASD requires an effective increase in the design factor of safety. This increase is to account for unknown dynamic effects. For the LRFD method such loads are accounted for by assigning a higher load factor. The designer should provide proper detailing and structural layout to minimize dynamic loading and cavitation. For example, proper arrangement of seal details minimizes vibration.

### 2-4. Inspection and Maintenance

HSS are often difficult to inspect and maintain due to poor access, particularly at submerged locations. Inspections should be performed in close contact with the inspected part; however, this is not always possible since HSS include submerged components which require dewatering for inspection. Where structures are difficult to inspect and maintain, guidance is provided in paragraph 3-4 for LRFD and paragraph 4-4 for ASD.

### 2-5. Deviations from Prescribed Design

Where special conditions exist, proposed modifications to the load and resistance factors or allowable stresses specified herein shall be submitted to CECW-ED for approval prior to completing feasibility phase work.

### 2-6. Commentary on Paragraph 2-2, Corrosion

*a. Introduction.*

(1) Paint systems specified in CW-09940 and EM 1110-2-3400 provide a high degree of protection. For underwater HSS requiring a higher degree of protection, cathodic protection (impressed current or galvanic systems) may be used to supplement the paint system. Impressed current systems for lock gates are often damaged and become inoperative if not carefully maintained; galvanic systems require less maintenance. However,

both systems require regular maintenance. If cathodic protection is included as part of the corrosion protection system, it is imperative that a long-term maintenance plan be developed, particularly for impressed current systems.

(2) General corrosion occurs uniformly over a large metallic surface. Specifying a uniform increase in design thickness is one means to protect a structure from this type of corrosion damage. However, the total structural cost is increased and the increase in member resistance to tension, compression, and bending effects is not uniform. The primary concern with corrosion damage in HSS is the occurrence of concentration cell corrosion, pitting corrosion, or galvanic corrosion.

(3) Concentration cell corrosion occurs at small local areas on metal surfaces which are in contact with water. Concentration cells can result from any number of differences in the environment, but the two most common are metal ion cells and oxygen cells. Either localized corrosion cell causes large tubercles of corrosion products to grow above the surface, generating a weak area in the steel member. Keeping the structure well painted and clean from mud deposits prevents this type of corrosion.

(4) Pitting corrosion is a form of extremely localized attack which results in small-diameter holes (in relation to their depth) to appear in the metal. This may be initiated by a material defect in the steel or a chip in the protective coating. Pitting corrosion is highly unpredictable since there is no means to identify where defects may occur. Regular inspection and maintenance practices can reduce the possibility of pitting corrosion.

(5) Galvanic corrosion is generally a result of current generated when two dissimilar metals are in contact and the two metals are in water.

*b. Requirements.*

(1) Kumar and Odeh (1989) recommend HSS be dry-blast cleaned to a grade approaching white metal grade for surface preparation prior to painting. Therefore, designers should detail the structure to allow sufficient room for the hose. Extra large drain holes located in areas where the sand may be trapped may be appropriate.

(2) Most HSS consist of welded construction. Using welded connections in lieu of bolted connections is advantageous when considering concentration cell corrosion. Areas on a surface in contact with an electrolyte having a high oxygen content are cathodic relative to those areas

where less oxygen is present. Localized areas where small volumes of stagnant solution may exist include sharp corners, spot welds, lap joints, and fasteners. Using butt welds instead of bolts; seal-welding lap joints; using continuous welds; and grinding weld splatter, slag, or any other deposits off the steel help to prevent concentration cell corrosion.

(3) Where dissimilar metals are used (generally carbon steel and stainless steel), the relative areas of each metal exposed are very important because the total amount of current that flows in the cell is dependent on the total area of both metals exposed. If the anode (carbon steel) is large with respect to the cathode (stainless steel), the current is distributed over a large area and the effect at each point will be slight. Conversely, if the cathode-to-anode ratio is large, the current becomes concentrated and severe corrosion can occur. If the carbon steel is painted and there is a small defect in the coating or it becomes damaged, then the relative areas have a large cathode-to-anode area and rapid corrosion can occur. Therefore, it is best to paint both surfaces. If the stainless steel coating has defects or damage, the current will not significantly increase even if the carbon steel has metal exposed. If the distance between the cathode and anode is large, resistance in the circuit will be sufficient to eliminate the galvanic corrosion problem.

## **2-7. Commentary on Paragraph 2-3, Dynamic Loading**

*a.* Dynamic loading that may occur in HSS is unpredictable in the sense that the dynamic forcing function is unknown. Unpredictable vibrations may be caused by imperfections in the operating machinery and guide slots, hydraulic flow, and load fluctuation due to passing ice. If the forcing function is known, a dynamic analysis can be used for design. At present, it is not feasible to define the load due to the many factors that affect such loadings and therefore special attention must be given to structure details. For example, supporting members of seals should maintain adequate stiffness to limit flexing which results in leakage and flow-induced vibration. The supporting members and arrangement of the bottom seal on a tainter gate can significantly affect its vibration due to flow conditions. Some of the structure types that have experienced vibration due to dynamic loading include tainter valves, vertical lift control gates, tainter gates, and miter gates.

*b.* Cavitation is also a concern where dynamic hydraulic loading occurs. Cavitation damage is a result of

unpredictable dynamic fluid action which causes extreme local negative pressures resulting in pitting and erosion of the surface. As for vibration, proper structure details and

good construction practices prevent cavitation from occurring.

## Chapter 3 Load and Resistance Factor Design

### 3-1. General

This chapter is intended to give a brief synopsis of LRFD methodology and to provide general guidance on LRFD for HSS. Appendixes B through I provide specific guidance and examples for different types of HSS. HSS designed by the LRFD method shall conform to guidance contained in AISC (1986), except as specified herein, and to the engineer manuals referenced in Appendixes B through I.

### 3-2. Design Basis

LRFD is a method of proportioning structures such that no applicable limit state is exceeded when the structure is subjected to all appropriate design load combinations. The basic safety check in LRFD may be expressed mathematically as

$$\sum \gamma_i Q_{ni} \leq \alpha \phi R_n \quad (2-1)$$

where

$\gamma_i$  = load factors that account for variability in loads to which they are assigned

$Q_{ni}$  = nominal (code-specified) load effects

$\alpha$  = reliability factor (see paragraph 3-4)

$\phi$  = resistance factor that reflects the uncertainty in the resistance for the particular limit state and, in a relative sense, the consequence of attaining the limit state.

$R_n$  = nominal resistance

The expression  $\sum \gamma_i Q_{ni}$  is the *required strength* and the product  $\alpha \phi R_n$  is the *design strength*. Load factors and load combinations for specific structure types are listed in the appropriate appendix.

### 3-3. Strength Requirements

Strength limit states are related to safety and load-carrying capacity (i.e., the limit states of plastic moment and buckling). Formulas giving the load combinations for

determining the required strength for buildings are given in American Society of Civil Engineers (ASCE) (1990) and AISC (1986). Similar load combinations pertaining to specific HSS are specified in Appendixes B through I. Structures shall have design strengths at all sections at least equal to the required strengths calculated for all combinations of factored loads and forces. The required strength of structural components shall be determined by structural analysis using appropriate factored load combinations. Each relevant limit state shall be considered. Elastic analysis is permitted unconditionally by this manual. Plastic analysis is permitted only with the approval of CECW-ED, and is subject to restrictions of paragraph A5.1 of AISC (1986).

### 3-4. Reliability Factors for HSS

For LRFD of HSS, resistance factors of AISC (1986) are multiplied by a reliability factor  $\alpha$ . The reliability factor  $\alpha$  shall be 0.9 except for the following structures where  $\alpha$  shall be 0.85:

a. For those HSS where inspection and maintenance are difficult because the HSS is normally submerged and removal of the HSS causes disruption of a larger project. Examples of this type of HSS include tainter valves and leaves of vertical lift gates which are normally submerged.

b. For those HSS in brackish water or seawater.

### 3-5. Serviceability Requirements

Serviceability is a state of acceptable performance in which the function of an HSS, its maintainability, durability, and operability are preserved under service or operating conditions. Serviceability should be maintained for the expected life of the project (typically 50 years for navigation and local flood protection projects and 100 years for other projects). The overall structure and the individual members, connections, and connectors shall be checked for serviceability. Limiting values of structural behavior (maximum deflections, vibrations, etc.) to ensure serviceability shall be chosen with due regard to the intended function of the structure. Serviceability may normally be checked using unfactored loads. The following limit states shall be considered in design for serviceability:

a. Deformation in the structural members and supports due to service loads shall not impair the operability or performance of the HSS.

b. Vibrations of the seals, equipment, or movable supports shall not impair the operability of the HSS.

c. Structural components shall be designed to tolerate corrosion or shall be protected against corrosion that may impair serviceability or operability of the structure during its design life. Closure provisions shall be made as required to maintain the structure.

### **3-6. Fatigue and Fracture Control**

a. *Fatigue requirements.* Fatigue design shall be in accordance with the provisions of Appendix K in AISC (1986) or AISC (1989) except as specified herein. The number and frequency of load cycles is a function of the HSS purpose and its environment. Determination of the total number of loading cycles shall consider known load fluctuations such as those due to operating cycles and fluctuations of hydraulic head. For certain HSS, vibration may result in unknown load magnitudes and number of cycles; therefore, a quantitative fatigue analysis is not possible. However, for HSS where vibration may produce significant cycles of stress, the choice of details shall be such to minimize susceptible fatigue damage (i.e., details with high fatigue resistance should be used where possible).

Welding processes induce significant residual stresses, and welded members may include high tensile residual stress in the welded region. Therefore, welded members which include any computed stress variation, whether it is tension or compression, shall be checked for fatigue. Deviation from this conservative assumption requires the approval of CECW-ED.

b. *Fracture control requirements.* For fracture-critical members (FCM) and/or components, the designer shall enforce controls on fabrication and inspection procedures to minimize initial defects and residual stresses, designate the appropriate temperature zone (see Table 3.1, Note 1), and specify the related minimum Charpy V-notch (CVN) fracture toughness. FCMs shall be defined as "members and their associated connections subjected to tensile stresses whose failure would cause the structure to be inoperable." Fracture critical members shall be identified by the designer (minimum requirements are given in Appendixes B through I). Minimum allowable CVN values shall be as given in Table 3.1. Tests to determine material CVN values shall be performed in accordance with the requirements of the American Association of State Highway and Transportation Officials (AASHTO) (1978). For construction of FCMs, fabricators, welding inspectors, and nondestructive examination personnel shall be certified

according to AASHTO (1978). Designers are referred to American Welding Society (AWS) (1990) and AASHTO (1978) for guidance on developing adequate quality control and fabrication procedures that will minimize initial defects.

### **3-7. Commentary on Paragraph 3-2, Design Basis**

Load factors and load combinations for structural steel design are based upon limit states of steel structures. Description of the methodology used in developing load factors and load combinations for buildings and other structures may be found in ASCE (1990), Ellingwood et al. (1982), Galambos et al. (1982), and McCormac (1990) and the commentary of AISC (1986). For HSS, the load and resistance factors are governed by items discussed in paragraph 3-8 (commentary of paragraph 3-4). The magnitude of a particular load factor is primarily a function of the characteristics (predictability and variability) of the load to which it is assigned and the conservatism with which the load is specified. A well known load with little variability or a conservatively specified load usually results in a relatively low load factor. Dead loads and static hydraulic loads are in this category. Transient loads are less known and, hence, they usually have a higher load factor.

### **3-8. Commentary on Paragraph 3-4, Reliability Factors for HSS**

Reliability factors are applied to AISC (1986) resistance factors for HSS design. This is to reflect a higher level of uncertainty (compared to building design) due to more aggressive environments in which HSS are placed. Historically, HSS have been designed using a higher factor of safety than that used for building design to account for the unpredictable nature of various items. The variables which require additional consideration for HSS include: facility of inspection; maintenance and repair or replacement (may require dewatering or submerged work by divers); possibility of corrosion (water may be fresh, polluted, brackish, or saline); economic considerations (loss of benefits due to shutdown of a larger project if replacement becomes necessary); possibility of severe vibrations or repeated stress reversals (hydraulic flow may cause vibrations and operating procedures may cause stress reversals); relative importance (HSS may be critical in the project operation); and design life of the structure in severe environments (50 to 100 years). For these reasons, reliability factors are applied to the resistance factors specified by AISC (1986) to effectively increase the factor of safety.

### 3-9. Commentary on Paragraph 3-6, Fatigue and Fracture Control

Fatigue damage and brittle fractures in HSS are rare but as structure designs, fabrication, and construction become more complex, the probability of brittle fracture increases. Welded construction, with its emphasis on monolithic structural members, increases the need to add fracture criteria to strength and buckling criteria when designing a structure. Various HSS have failed due to fatigue and brittle fracture. Many of the cracking problems that have occurred in HSS originate from poor weld details or poor fabrication. For control of fatigue and fracture, consideration must be given to the following parameters: (a) stress range, detailing, and the number and frequency of load cycles to control fatigue and (b) geometry, toughness, and stress levels to control fracture.

#### *a. Fatigue requirements.*

(1) Fatigue is the process of formation and growth of a crack due to repeated fluctuating loads. The designer cannot control the number and frequency of load cycles since this is a function of the operational requirements of the HSS. However, design options include selection of larger members to control the stress range and choice of details with low stress concentrations which have a high fatigue life.

(2) Significant vibration may occur in certain HSS due to hydraulic flow, imperfect seals, movable supports and operating machinery, and impact of passing ice or debris which may occur during a single operating cycle. For these situations, the magnitude of load and the number of load cycles are unknown. Unless predictions for load magnitude and frequency may be made using probabilistic methods, a quantitative fatigue analysis is not possible. However, the possibility of fatigue damage can be controlled by considering the design options given in the previous paragraph.

(3) AISC (1986, 1989) do not require any fatigue check for members with a calculated repetitive stress variation from zero to compression, since crack propagation will not occur in the absence of tensile stress. However, whether a stress variation is tensile or compressive, paragraph 3-6a does require a fatigue check for welded members. This is due to the possible presence of large residual tensile stresses caused by welding processes. For example, if a residual tensile stress of 25 ksi exists, a calculated stress variation from zero to -10 ksi would actually be a variation from 25 ksi to 15 ksi, which could cause fatigue cracking. Tensile residual stresses for

welded members are near the yield stress in most cases. The consideration of residual tensile stress is a conservative assumption for fatigue design. It is not currently a uniform practice in the United States; however, it is common in Europe. The assumption is currently favored by many welding specialists.

#### *b. Fracture control requirements.*

(1) Fracture is the sudden growth of a crack which may cause failure of a component. Fracture behavior is governed mainly by nominal stress level, material toughness, and geometry of the existing crack or flaw. The fracture control requirements specified herein are based on imposing material toughness requirements and limiting geometry of initial flaws for FCMs, the most critical structural components. Fracture toughness criteria are supplemented with welding and inspection requirements to form a complete fracture control plan. The toughness is controlled by imposing minimum CVN requirements per Table 3-1 and the geometry of initial flaws is controlled by imposing strict fabrication and inspection requirements. Project specifications should require qualification of fabricators and welding inspectors according to AASHTO (1978), to assure that FCMs and their components are in compliance with the requirements specified in paragraph 3-6.

(2) Table 3-1 values are the same as those required by AASHTO (1978) for steel bridges. The basic requirement used in the development of Table 3-1 was to ensure elastic-plastic behavior (i.e. prevent brittle fracture) under service loading at the minimum operating temperature. CVN tests were carried out under service load rates to determine the minimum CVN requirements to assure elastic-plastic behavior for various service temperatures (AASHTO 1978).

(3) Material toughness is affected by load rate, yield strength, service temperature, component thickness, and type of detail. Each of these effects was considered in the development of Table 3-1, and all but load rate are explicitly accounted for in Table 3-1. The following discussion is included to provide a brief explanation of toughness requirements for the various categories of Table 3-1. A more complete discussion is provided in AASHTO (1978) and Barsom and Rolfe (1987).

*(a) Load rate.* The effect of load rate was considered in the determination of required test temperatures. A consistent temperature shift exists between CVN values obtained for specimens subject to a given load rate (less than impact load rate) and those obtained for impact

**Table 3-1**  
**Fracture Toughness Requirements for Fracture Critical Members**

Welded or Mechanically Fastened	Grade $\sigma_{ys}$ (ksi)	Thickness (in.)	Zone 1 (ft-lb at °F)	Zone 2 (ft-lb at °F)	Zone 3 (ft-lb at °F)
Welded	36	$t \leq 1.5$	25 at 70	25 at 40	25 at 10
		$1.5 < t \leq 4.0$	25 at 70	25 at 40	25 at -10
Welded	50	$t \leq 1.5$	25 at 70	25 at 40	25 at 10
		$1.5 < t \leq 2.0$	25 at 70	25 at 40	25 at -10
		$2.0 < t \leq 4.0$	30 at 70	30 at 40	30 at -10
Welded	70	$t \leq 1.5$	30 at 20	30 at 20	30 at -10
		$1.5 < t \leq 2.5$	30 at 20	30 at 20	30 at -30
		$2.5 < t \leq 4.0$	35 at 20	35 at 20	35 at -30
Welded	100	$t \leq 2.5$	35 at 0	35 at 0	35 at -30
		$2.5 < t \leq 4.0$	45 at 0	45 at 0	Not allowed
Mechanically Fastened	36	$t \leq 1.5$	25 at 70	25 at 40	25 at 10
		$1.5 < t \leq 4.0$	25 at 70	25 at 40	25 at -10
Mechanically Fastened	50	$t \leq 1.5$	25 at 70	25 at 40	25 at 10
		$1.5 < t \leq 4.0$	25 at 70	25 at 40	25 at -10
Mechanically Fastened	70	$t \leq 1.5$	30 at 20	30 at 20	30 at -10
		$1.5 < t \leq 4.0$	30 at 20	30 at 20	30 at -30
Mechanically Fastened	100	$t \leq 4.0$	35 at 0	35 at 0	35 at -30

NOTE:

1. Zone 1 minimum service temperature is 0°F and above; Zone 2 minimum service temperature is from -1°F to -30°F; and Zone 3 minimum service temperature is from -31° to -60°F.
2. Charpy impact tests are required on each end of each piece tested for Zone 3.

specimens. The CVN value for a specimen tested under a service load rate at service temperature is equivalent to the CVN impact value for a specimen tested at a temperature which is a constant magnitude greater (temperature shift) than the service temperature. For example (see Table 3-1), for welded 36-ksi components of thickness less than 1.5 in. which are subject to bridge service load rates and minimum service temperature, ductile behavior is assured if CVN impact values are at least 25 ft-lb for tests conducted at 70°F higher than the minimum service temperature. The temperature shift is dependent on service load rate. The temperature shift comparing static and impact load rates is maximum and as load rate increases, the temperature shift decreases. Adoption of bridge criteria for HSS is generally conservative since loading rates on bridges are likely higher than those which occur on most HSS.

(b) *Yield strength.* The more stringent requirements for steels of higher yield strengths are identified by higher CVN requirements and lower test temperatures. The higher CVN requirements for increased yield strengths are due to the fact that the design stress is generally higher which will result in more elastic stored energy. In order to attain the same degree of safety as in the lower yield steels, the CVN requirement is also increased. The reduced test temperatures are based primarily on the fact that the temperature shift between toughness under service load and impact load decreases with increasing yield strength; thus, lower CVN impact test temperatures are specified to reflect the decrease in temperature shift.

(c) *Service temperature.* The expected service temperature for a structure is a critical factor in determining toughness requirements since most steels exhibit a

transition from ductile to brittle behavior at a certain temperature. As temperature decreases, toughness and ductility decrease. Therefore, for lower minimum service temperatures, CVN specimens must be tested at lower temperatures to ensure that the steel has adequate toughness.

(d) *Component thickness.* For thick plates under tensile loading, through-thickness stresses at a crack tip are large due to the through-thickness constraint. This results in a triaxial stress state which reduces the apparent ductility of the steel by decreasing the shear stresses. Because yielding is restricted, the constraint ahead of the notch is increased resulting in reduced toughness. In order to assure ductile behavior, the CVN requirements of Table 3-1 are increased for increasing thickness.

(e) *Detail.* Welded details require more conservative CVN values than mechanically fastened details for certain thicknesses and service temperatures. The heat input due to welding can reduce toughness properties in the heat affected zone (HAZ). The HAZ is the area of unmelted parent material adjacent to the weld, which is sufficiently heated by the welding that its metallurgical properties are affected. This area may be of special importance in thick members since these usually have lower toughness and are subject to greater heat input during welding. Unfortunately, stress concentrations often overlap the HAZ of welds, thus combining the adverse effects of high stress and low toughness.

## Chapter 4 Allowable Stress Design

### 4-1. General

HSS designed by the ASD method shall conform to specifications contained in AISC (1989), except as specified herein, and to the engineer manuals referenced in Appendixes B through I.

### 4-2. Design Basis

ASD is a method of proportioning structures such that allowable stresses are not exceeded when the structure is subjected to specified working loads. An elastically computed stress is compared to an allowable stress as represented by

$$f(\sum Q_i) \leq F_{\text{allow}} \quad (3-1)$$

where

$f(\sum Q_i)$  = elastically computed stress arising from the appropriately combined nominal loads

$F_{\text{allow}}$  = allowable stress (yield stress, buckling stress, shear, net section tension, bearing strength, etc. divided by a factor of safety).

### 4-3. Load and Stress Requirements

*a. Loads.* Loads are divided into Group I and Group II loadings as follows:

Group I	
Dead load	Buoyancy load
Live load (serviceway)	Hydrostatic load
Thermal stress load	Operating equipment load
Ice loads (static)	
Group II	
Impact (vessel, debris, ice)	Water hammer
Wind loads	Ice loads (transient)
Wave loads	Operational basis earthquake (OBE)

(1) Ice loads may be considered as Group I (static load) or Group II (impact; short duration load) loads depending on circumstances.

(2) When the loading includes Group II loads acting alone or in combination with Group I loads, allowable stresses may be increased 1/3 above the values otherwise provided. However, the section thus provided shall not be less than that required for Group I loads when designed with the normal allowable stresses.

*b. Stresses.* It is considered necessary to reduce the allowable stresses given in AISC (1989) for HSS design (see commentary for paragraph 4-4 (paragraph 4-8)). Allowable stresses for three main types of HSS are specified in paragraph 4-4. Examples of each HSS type are discussed in the Commentary. If a structure has characteristics of more than one type, the lesser allowable stress is required.

### 4-4. HSS Types: Modifications for Allowable Stresses.

*a. Type A.* HSS which are used for emergency closures and which are subject to severe dynamic (hydraulic) loading or are normally submerged where maintenance is difficult, and removal of the HSS causes disruption of the project. For Type A HSS, the allowable stress shall be 0.75 times that allowed by AISC (1989).

*b. Type B.* HSS which are normally hydraulically loaded and are not subjected to unknown dynamic loading. For Type B HSS, the allowable stress shall be 0.83 times that allowed by AISC (1989).

*c. Type C.* HSS which are used for maintenance and are not considered emergency closures. For Type C HSS, the allowable stress shall be 1.1 times that allowed by AISC (1989). These allowable stresses are the maximum allowable values and may not be further increased due to Group II loading.

### 4-5. Serviceability Requirements

Guidance in paragraph 3-5 is applicable.

### 4-6. Fatigue and Fracture Control

Guidance in paragraph 3-6 is applicable.

#### 4-7. Commentary on Paragraph 4-3, Load and Stress Requirements

*a.* ASD guidance for HSS considers Groups I and II loading, and Types A, B, and C stresses. The loading groups determine which conditions must stay within the modified AISC allowable stresses and which loading conditions are permitted a 1/3 increase in allowable stress. Because of the environment in which HSS are placed, modifications to AISC allowable stresses for HSS types are applied to increase the factor of safety above that which is used in building design.

*b.* Group I loads include those loads which are relatively constant for a significant time period, and Group II loads are those which vary with time. The 1/3 increase in allowable stress for structures subject to Group II loads acting alone or in combination with Group I loads is to account for the improbability of the simultaneous occurrence of maximum lifetime loads. Ice loads may be considered either Group I or Group II depending on the circumstances. If ice hanging on the structure is being considered as additional dead load or it is applying a lateral force due to expansion from thermal effects, it is considered a Group I load. If ice is acting dynamically on the structure due to wind or flowing water, it is considered a Group II load.

#### 4-8. Commentary on Paragraph 4-4, HSS Types: Modifications for Allowable Stresses

*a.* In general, it is considered that HSS are subjected to more extreme environments and are subject to less predictable loads than are buildings. Variables listed in paragraph 3-8 (commentary of paragraph 3-4) are among the causes of this additional uncertainty. Therefore, an increase in the design factor of safety over that used for building design is considered necessary for HSS design.

*b.* The grouping by HSS type is a means to distinguish characteristics of different HSS. Type A is

considered to be the most extreme case, and Type C the least extreme case.

*c.* Type A includes those structures which are subject to unpredictable dynamic loading, or those which are normally submerged where maintenance is difficult. Unpredictable dynamic loading may occur as a result of hydraulic fluctuations in velocity and pressure due to abrupt changes in structure geometry or gate position as it is operated. Severe, unpredictable vibrations may also occur on structures subject to significant amounts of passing ice. Type A HSS include emergency gates, regulating gates where the structure passes through moving water under full pressure and flow conditions (unpredictable dynamic loading may occur), tainter and vertical lift crest gates used for regulation and subject to unknown dynamic hydraulic forces, and lock valves (normally submerged and difficult to maintain).

*d.* Type B includes structures for which dynamic loading is not significant and maintenance and inspection can be performed on a regular basis. HSS that may be classified as Type B include tainter crest gates, vertical lift crest gates, power intake gates designed for top of power pool, lock gates (miter gates, lift gates, and sector gates), and floodwall closures.

*e.* Type C structures include temporary closure items which are used to dewater for maintenance or inspection of gates, gate slots, and draft tubes. Stoplogs, bulkheads, draft tube gates, and bulkhead gates are included in this type. Such structures are not considered emergency closures and are usually opened and closed under balanced head conditions. The 1.1 factor applied to AISC (1989) allowable stresses reflects a 1/3 increase of the Type B allowable stresses. This increase is considered appropriate due to the fact that such structures are used on a temporary basis under essentially constant loading.

## Chapter 5 Connections and Details

### 5-1. General

Connections consist of connecting elements (e.g., stiffeners, gusset plates, angles, brackets) and connectors (bolts, welds, or for older HSS, rivets). Connection design shall conform to the specifications contained in AISC (1986, 1989) and AWS (1990) except as specified herein. Critical connections should be fully detailed by the design engineer. Connections which are considered noncritical may be detailed by the fabricator; however, the designer shall clearly define the requirements of the noncritical connection. Any deviation from details originally specified by the design engineer shall be reviewed and approved by the design engineer. Details that will result in safe economical fabrication methods shall be used. Special critical connections for specific structure types are discussed in the appropriate appendixes.

### 5-2. Design Considerations

Connections shall be designed to transfer the required forces obtained from the structural analysis, and shall maintain sufficient ductility and rotation capacity to satisfy the particular design assumption. Connection designs must consider stress concentrations, eccentricities, field splices, imposed restraints (fixity), and fatigue resistance. Following is a discussion of these design considerations.

*a. Stress concentrations.* Avoid abrupt transitions in thickness or width, sharp corners, notches, and other stress raising conditions.

*b. Eccentricities.* Effects of eccentricity of fastener groups and intersecting members shall be accounted for in the design of connections (see Chapter J of AISC (1986, 1989)).

*c. Splices.* Shipping restrictions require large HSS to be delivered in sections, which makes field splicing necessary to form the completed structure. Splices should be located in uncongested areas of low or moderate stress. When splices are necessary, they should be shown on the drawings with accompanying splice details or design forces.

*d. Restraints.* Connections between intersecting members are usually designed to be rigid (original angle

between connected members remains fixed) or simple (pinned). If the design assumed a pinned connection, the as-built connection should provide for members to rotate relative to each other to accommodate simple beam end rotation (to accomplish this, inelastic deformation is permitted).

*e. Fatigue.* Connections shall be designed to minimize the possibility of fatigue damage by using proper detailing practices (see AISC (1984, 1986, 1989) and AASHTO (1978)), and limiting the stress range in accordance with Appendix K of AISC (1986, 1989). Corrosion-fatigue shall be controlled with a well designed and maintained corrosion protection system.

### 5-3. Bolted Connections

Fully tensioned high-strength bolts shall be used for all HSS structural applications. For nonstructural applications, use of A307 bolts or snug-tight high-strength bolts is allowed, provided requirements of AISC (1986, 1989) are followed. Bolts shall be proportioned for the sum of the external load and tension resulting from prying action produced by deformation of the connected parts. AISC (1984, 1986, 1989) and Kulak, Fisher, and Struik (1987) are useful aids to designing bolted connections.

### 5-4. Welded Connections

Most HSS are constructed using welded connections. AISC (1984, 1986, 1989) and AWS (1990) are useful aids to selecting the connection details. Welding requirements of AISC (1986, 1989) and AWS (1990) shall be followed. Thick plate weldments shall be designed considering heat requirements (see Section 4 of AWS (1990)), toughness requirements, and geometric requirements (see Section A3 of AISC (1986, 1989) for toughness and geometric requirements). Intersecting and overlapping welds should be avoided. Intermittent welds should be avoided for dynamically loaded members and members subject to corrosion. Through-thickness welds should have backing bars removed and should be ground smooth. The designer shall review and approve the contractor's proposed welding processes and shop drawings.

### 5-5. Commentary on Paragraph 5-1, General

Connections for HSS are usually in a more severe environment than connections for buildings. HSS connections may be exposed to weather, fresh or salt water, flowing water, and, for many HSS, impacts. AISC (1986 or 1989) can be used as guidance but should be

supplemented with AASHTO (1989) since many HSS members have more in common with bridges (sizes, types of connections, and loads) than with steel building frames. Connection details must be consistent with the assumptions used in the design analysis of the structure and must be capable of transferring the required forces between connected members. The forces may consist of any combination of axial or shear loads and bending or torsional moments. Connections may also provide stiffness to limit relative movement between members. Most HSS use welded or bolted connections; however, many older structures have riveted connections.

#### 5-6. Commentary on Paragraph 5-2, Design Considerations

*a. Stress concentrations.* Stress concentrations in connections are often ignored in design with no decrease in load-carrying capacity. This is because ductility of the steel redistributes localized high stresses. However, this does not mean details that cause stress concentrations can be ignored. Attention should be given to areas of large change in cross section such as termination of cover plates, welds where backing bars have not been removed, and at sharp discontinuities. These details are critical for fatigue resistance. AWS (1990) shows geometries for welded connections that minimize stress concentrations at transitions between members of different thicknesses or widths.

*b. Eccentricities.*

(1) Axial loads eccentric from fastener group centroids can significantly increase local stresses or individual fastener loads due to additional shear and bending imposed by the eccentricity. While eccentricities in statically loaded single-angle, double-angle, and similar members may be of minor consequence, connections for members subject to cyclic loading should be balanced about their gravity axes; if not, provision shall be made for bending and shearing stresses due to the eccentricity.

(2) The designer has the option of selecting a concentric connection or, in some cases, an eccentric connection. A concentric connection is detailed so that the gravity axes of all members framing into the connection pass through a common point. This ensures that the axial force in an intersecting member does not produce an additional moment in the connection. However, in some cases a concentric connection may be undesirable because it can require poorly shaped elements such as long gusset plates with a limited buckling capacity that is difficult to assess.

(3) An eccentric connection may be detailed to simplify the design of gusset plates. For example, a member may be located such that its line of force passes through the corner of the gusset plate. However, the lines of action of the force in the intersecting members usually do not pass through the same point. The axial force acting eccentrically will produce a moment in the connection which must be distributed among the connected members based on their relative stiffness. See AISC (1984) for illustrated examples.

#### 5-7. Commentary on Paragraph 5-3, Bolted Connections

In the past many HSS have used riveted connections; however, the use of rivets has largely been replaced by use of high strength bolts. Per AISC (1986, 1989), full tightening is required for cyclic loads, for bolts in over-size holes, and when it is necessary to improve water tightness, or if corrosion of the joint is a concern. Therefore, for all HSS structural applications, fully tensioned high-strength bolts shall be used. Bolted connections are much less common on HSS than on buildings or bridges. Typically, bolted connections for HSS are limited to machinery and appurtenances, splices, sill plates, thick plates or jumbo sections (over 1.5 in. thick), steel members embedded in or supported by concrete, locations where future adjustments may be required, or elements that may need replacing sometime during the life of the structure.

#### 5-8. Commentary on Paragraph 5-4, Welded Connections

Many HSS contain thick (greater than 1.5 in. thick) plate weldments. Critical connections on HSS often consist of full penetration or large fillet welds to develop the full strength of a part. Heavy welding is labor intensive and may result in member distortion and large residual stresses. Thick plates and jumbo rolled shapes often exhibit low toughness away from rolled surfaces, and lamellar discontinuities are more prevalent than in thinner plates. Thermal effects due to welding further decrease material toughness and produce high residual stresses which act on these low toughness areas and lamellar discontinuities creating high potential for cracking. The adverse thermal effects are reduced with gradual heating and cooling of the weldment as it is welded, and proper selection of weld process and procedures. Residual stresses in weldments are increased with increasing external constraint so the designer should detail connections to minimize constraint.